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The Performance of Turlough Hill Upper Reservoir

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THE PERFORMANCE OF TURLOUGH HILL UPPER RESERVOIR

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ABSTRACT

The upper reservoir of the Turlough Hill pumped storage station was commissioned by the Irish Electricity Supply Board in 1974. This upper reservoir comprises a rockfill embankment within which there is a useful capacity of 2.3 million cubic metres. It was constructed in difficult conditions on a very isolated and exposed site at the top of a mountain, using rockfill derived from the local granitic bedrock. The reservoir is lined with asphaltic concrete.

The paper describes briefly the design and construction of the embankment. It then summarises the results of monitoring and inspecting, with particular emphasis on movement, seepage and on the performance of the asphaltic concrete lining.

The reservoir has performed very well in its 23 years of service to date. Some early seepage problems were found to be due to an isolated defect in the lining. In recent times a slight increase in the quantity of measured seepage has been noted. The amount of seepage is small and is most likely due to the natural aging of the bitumen in the lining system on exposure to sunlight. The paper illustrates how most of the problems are located on the sides of the reservoir, which are exposed to the evening sun.

The paper will conclude with a summary of the measures being adopted to ensure the continued good performance of the reservoir into the next century.

KEYWORDS

Dam, Embankment, Movement, Seepage, Asphalt, Bitumen

INTRODUCTION

The 292MW Turlough Hill pumped storage station is located in County Wicklow in the Republic of Ireland. It was commissioned by the Irish Electricity Supply Board in 1974. A lower reservoir was readily available in the form of a natural lake. A principal feature of the development of the scheme was the design and construction of the upper storage reservoir, which has a capacity of 2.3×10^6 m^3.

It comprises a soil and rockfill embankment and was constructed in difficult conditions on a very isolated and exposed site at the top of a mountain, using locally derived rockfill. The reservoir is lined with asphaltic concrete.
This paper will describe briefly the design and construction of the embankment. It will summarise the results of monitoring and inspecting of the embankment, with particular emphasis on seepage and will discuss the reasons for some of the minor problems which have occurred. The paper will conclude by summarising the measures being adopted to ensure the continued good performance of the reservoir into the future.

THE SITE

The site is located 8km from the village of Laragh, in the Wicklow Mountains, some 60km south of Dublin, Ireland. The site location is shown on Figure 1. The site is very exposed and isolated, being on a peat-covered plateau of a mountain at an elevation of some 670mOD. As well as being located on a natural plateau, the area was ideal for the development of a pumped storage scheme, as a lower reservoir was readily available in the form of a natural lake at the foot of the plateau.

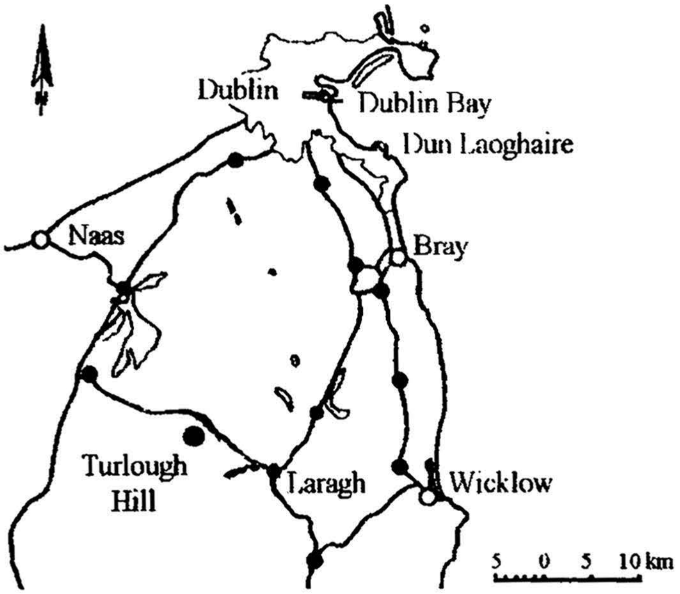


Fig. 1 Site location

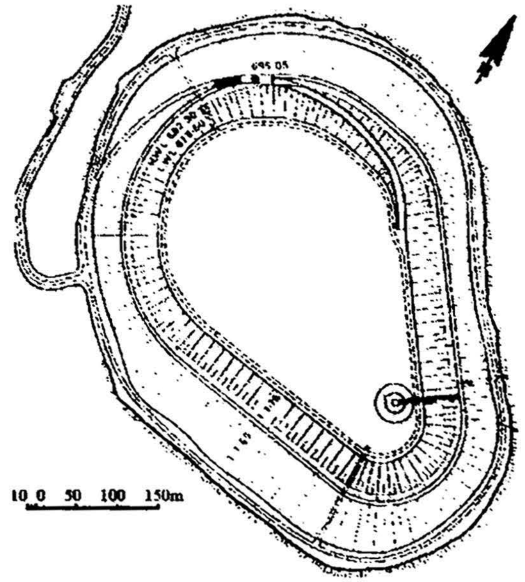


Fig. 2 Plan layout of embankment

GEOLOGY

The scheme is located within the Leinster Granite, which in the Turlough Hill area is a strong uniform coarse grained quartz diorite and contains the usual granite associated minerals including calcite. The rock is overlain by 1m to 4m of peat. On excavation of the peat the bedrock was found to be intensely altered, with the ratio of sound to decayed rock being of the order of 25:75. The alteration is thought to be due to the leaching of calcite, and was of greater extent than had been predicted by the site investigation. The geology of the area is also complicated by some significant fault zones, up to 15m wide. The geology of the site has been described in detail by Knill (1972) and by Keenan (1973).

UPPER RESERVOIR DETAILS

The upper reservoir, which has a useful capacity of 2.3 x 10^6 m^3, comprises a ring shaped soil and rockfill embankment. The main features of the embankment are as follows:

Maximum height:	34m
Crest length:	1445m
Slopes, inner face:	1 on 1.75
Slopes, outer face:	1 on 1.85
Volume of fill:	1,300,000 m^3

The plan layout of the embankment is shown on Figure 2. A cross section through the embankment is shown on Figure 3.

Embankment Fill

It had been originally intended to construct the reservoir embankment in a series of zones using different grades of rockfill, all excavated from the underlying bedrock. However, the weathered nature of the blasted and excavated material prevented this. The embankment now comprises an approximate homogenous mass of uniformly graded material with particle sizes ranging from silt size to a maximum of 500mm and has a permeability of about 1 x 10^{-6} m/s.

Drainage

The purpose of the drainage layer on the water side slopes is to provide a stable surface for the reservoir lining and to ensure that leakage through the lining does not cause a build up in pore pressure under rapid drawdown rates. It comprises a 1.2m thick layer of 10mm to 200mm stone with a surface blinding layer of 10mm to 55mm stone. The reservoir floor drainage layer is of 10mm to 55mm stone, 200mm thick.

A ring drainage gallery is provided at the inner toe of the embankment to monitor seepage through the reservoir lining, see Figure 3. Both sides of the gallery are connected to the drainage layers by means of pipes at 2m horizontal intervals.

Lining

The reservoir is lined with an asphaltic concrete lining. It was the first use of asphaltic concrete for such a purpose in Ireland. The main features of the specification for the lining as regards waterproofing were as follows:

- it should bridge any cavities occurring in the drainage layer and accommodate movement of the embankment,
- it should be stable at temperatures between -20°C and 70°C,
- the seepage from the reservoir should not exceed 6 l/s,
- the permeability value should not exceed 1x10^{-11} m/s
- it should be guaranteed for 5 years.

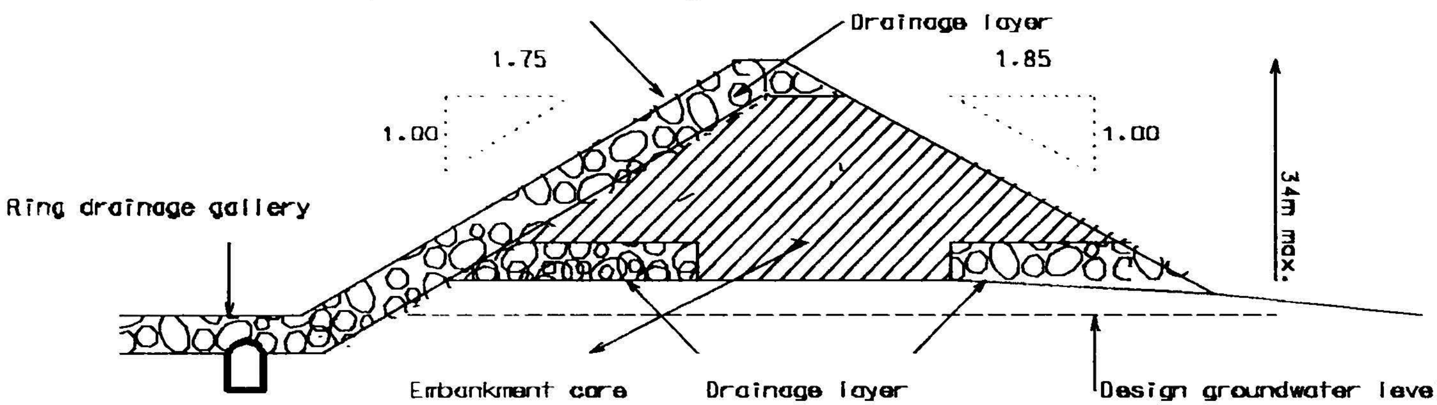


Fig. 3 Cross section of embankment

So as to meet the requirements of the specification, the lining system adopted comprised of two asphalt layers, a lower binder layer, to bridge any cavities in the drainage layer, and an upper dense layer. The details of the lining for the embankment sides and floor are shown on Figure 4. The key component in each case is the 50mm to 60mm thick layer of dense asphaltic concrete. The material for this layer comprised 46% crushed diorite from 2mm to 12mm, 41% of sand, 12% of calcium carbonate filler, 1% asbestos (slopes only) and bitumen amounting to 7% of the total mix. The diorite was sourced, 35km to the south, at Arklow.

DESIGN

Based on early experiences on site regarding excavation of the rock, the design philosophy changed from one of an embankment comprising various zones to one in which the overall aim was to produce a well mixed general fill and an embankment in which there were minimal voids and no bridging between rocks.

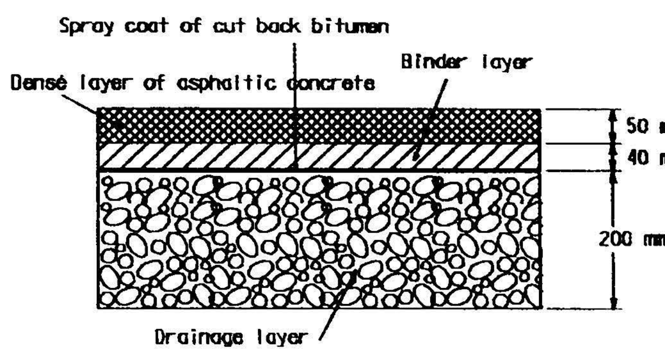
Slope stability analyses were carried out using conventional limit state slip circle analyses. The key design parameter is the angle of internal friction (ϕ^1) of the fill. It was conservatively assumed that ϕ^1 was governed by that of the fine fraction and was obtained by removing the particles greater than 10mm in size and then carrying out drained triaxial tests. It was found that there was a 90% probability of the ϕ^1 value exceeding 33° and this value was used in design. It was recognized that the presence of larger particles in the fill meant that in reality the ϕ^1 value was closer to 40°. The unit weight of the fill was assumed to be 2t/m^3 as measured by in-situ density tests. The stability analyses yielded factors of safety of 1.4 and 1.3 for the inner and outer slopes respectively. These values reduced to 1.0 when seismic loading was assumed.

CONSTRUCTION

The construction of the embankments is described in detail by Fives (1975) and Water Power (1972). Excavation of the fill material was carried out by a combination of blasting and ripping from the reservoir floor and from a nearby quarry. The material was crushed, to give a maximum particle size of 500mm, and then placed and compacted in place using conventional earthworks plant to 95% of the modified Proctor density. Quality control was by means of compaction trials, in-situ density, permeability and plate loading tests as well as laboratory grading, compaction, strength and permeability tests.

The lining material was first batched ensuring that the maximum temperature was limited to 190°C. The material was placed using conventional paving machines. A winched rubber wheeled machine was used on the slopes. The minimum placing temperature was 160°C. Compaction was achieved, at a minimum temperature of 130°C by a combination of a winched 2.2t roller and a 1.2t vibrating roller hauled by the paving machine.

A special vibrating shoe was used to compact the leading edge of the strip. The machines were fitted with infra red heaters to reheat the edge of the previously laid strip. Rolling was arranged to compact towards the joints, the aim being to achieve a seamless lining. This was not easily achieved in the weather conditions and it ultimately proved necessary to treat each joint by reheating and tamping with a vibratory tamper. Quality control of the placed asphalt, particularly at joints, comprised mostly of vacuum tests to assess permeability.



Floor detail (NB side detail differs only in drainage layer – 1.2m thick).

Fig. 4 Asphaltic concrete lining

MONITORING SYSTEM AND RESULTS

The monitoring system of the completed embankment / lining in operation consisted of instruments to measure movement, water level soundings and seepage.

Movement - Outer Slope Position

Vertical and horizontal movement of the embankment outer slopes was monitored, at four cross sections, by the use of a probe to locate aluminum plates along the outside of a plastic pipe built into the embankment. This system was known as the Dr. Idel movement monitoring system and was considered to be the state of art at the time. Measurements taken at an early stage showed no measurable movement. However, problems soon arose with the pulley and wire system used to pull the electronic probe and with dealing with joints on the plastic pipe. All of the instruments soon became unusable.

The datum points from the Dr. Idel system were subsequently used for monitoring, by triangulation using a theodolite from instrument stations founded on rock.

A typical result of the triangulation surveys for Cross Section 7 on the western side of the embankment is shown on Figure 5. The readings are given in terms of deviation from the original reading in 1973. It can be seen that the movements measured are negligible and are within the accuracy of the surveying equipment.

Movement - Horizontal Crest Position

Surveying is carried out from the embankment crest to assess the relative horizontal movement of various points on a crest to other points opposite. The results of the surveying of horizontal crest position are show relative movements of between 0mm and 20mm, with no discernible pattern. Again it can be said that the movement is negligible and is within the surveying accuracy.

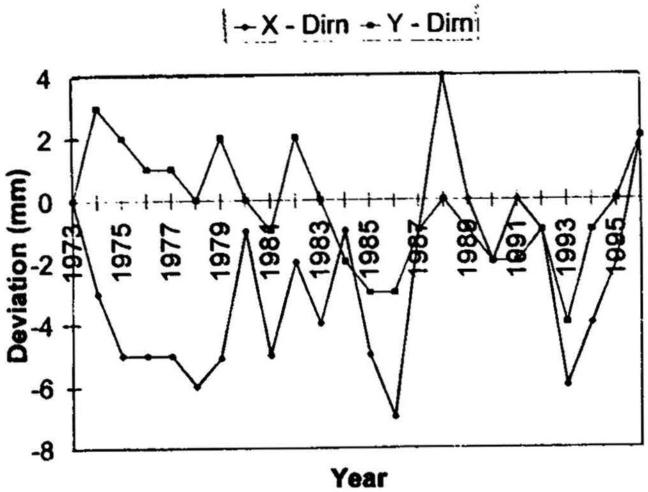


Fig. 5 Results of triangulation surveying C/S 7

Movement - Crest Leveling

Leveling of the embankment crest is carried out using conventional surveying with a precise level at points at 50m intervals around the crest.

The results of crest leveling for three points (7/1 to 7/3) on the southern side of the embankment are shown on Figure 6. The readings are given in terms of the deviation from the original reading in 1973. The trend is an initial settlement of some 20mm over the first 3 years followed by a gradual heave of the crest. These movement are possibly due to thermal effects and are of no significant consequence.

Water Level Soundings

Water level sounding readings are taken from 9 boreholes located on the outside toe of the embankment and from 5 points at the base of the ring drainage gallery. The results of monitoring of Gallery Well 3 and the adjacent outer well at Cross Section 37 are shown on Figure 7. The readings follow a similar trend with an initial fall in level followed by a gradual rise since 1984. The increase in head outside the embankment has been greater than that inside the gallery. This same pattern is repeated elsewhere and may indicate a trend for external groundwater to flow into the ring drainage gallery.

Seepage

Seepage is read at each of the pipes located at 2m centres around the drainage gallery. Total seepage is measured at the outlet of the ring drainage gallery. An alarm electrode is present at this location and was originally set to sound if total flows exceed a seepage of 1 l/s.

Following initial filling and regularly during the contractual 5 year maintenance period, seepages of about 5 l/s were recorded. This seepage was eventually traced to a crack in the lining system which was repaired in 1975, see next section.

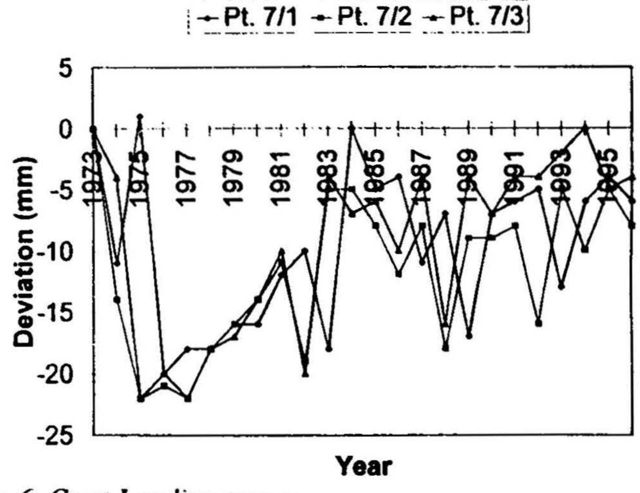


Fig. 6 Crest Leveling survey

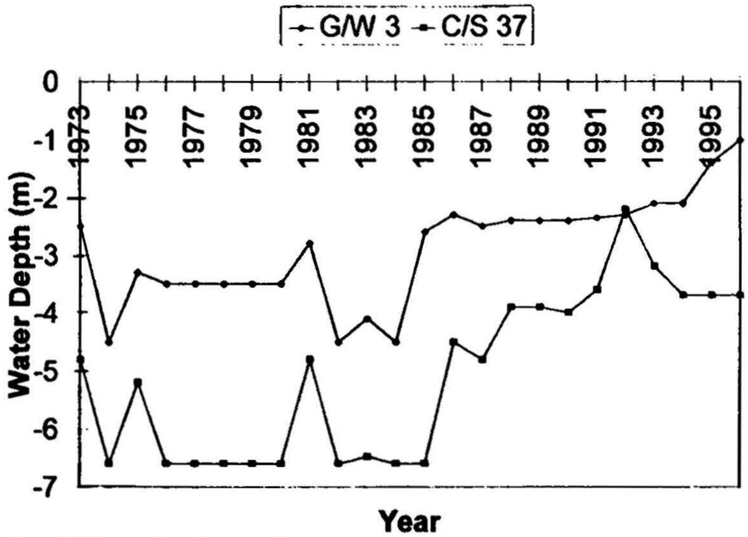


Fig. 7 Water level soundings

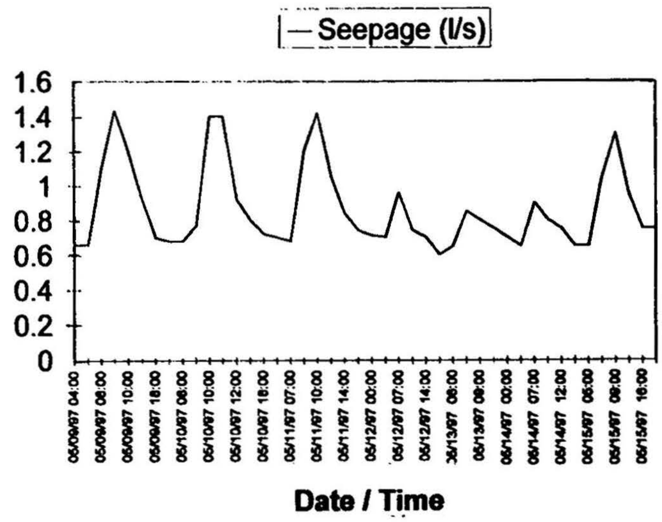


Fig. 8 Total seepage w/c 9th. May 1997

Between 1975 and 1990 seepage quantities were negligible. Between 1990 and 1995, a slight increase in seepage, up to 0.25 l/s was noted. This seepage was from a temporary works drainage pipe, which was subsequently built into the drainage gallery and was mainly seasonal in nature.

Since December 1995 a further increase in seepage quantities has been noted. The alarm system has been triggered at least three times by seepages in excess of 1l/s. Maximum seepage values of 3 l/s have been noted and the alarm system has been reset to trigger at a seepage of 4 l/s.

A typical record of seepage rate for a week in May 1997 is shown on Figure 8. The pattern shows peak seepage early in the morning with the seepage reducing during the day. This corresponds directly with reservoir water level, which is at its highest in the early morning, following overnight low cost pumping from the lower reservoir, and which falls during the day as electricity is generated.

The seepage is mostly from a series of pipes on the eastern side of the reservoir, as shown on Figure 9, from the pipes located on the outside face of the drainage gallery, suggesting that the seepage originates from the sides of the reservoir.

Experience shows that seepage increases if the reservoir is maintained at a high level for prolonged periods, e.g. holiday weekends. There is roughly a 4 hour lag between maximum reservoir level being attained and peak seepage. Seepage appears to be independent of climatic or seasonal effects, suggesting that the contribution from the external groundwater is very small.

PERFORMANCE OF LINING SYSTEM

The performance of the asphaltic concrete lining is examined by inspection during periods of low water level in the reservoir, as required for maintenance purposes.

General Performance of Bitumen Systems

Bitumen is well known to deteriorate with time, particularly under the influence of sunlight. The deterioration is due to an aging process, where hardening occurs due to oxidation. Typical design lives for systems similar to that at Turlough Hill would be 20 to 30 years (Shell, 1991))

Crack on Floor (1975)

The most significant defect in the lining system was a 10mm wide 4m long discontinuous crack which was discovered and repaired in 1975. The location of this crack is shown on Figure 9. This crack caused the high early seepages discussed in the previous section. No reason for the crack was discovered. It was repaired by removing the defective lining material to the drainage layer and replacing it by 3 layers of wearing course asphalt.

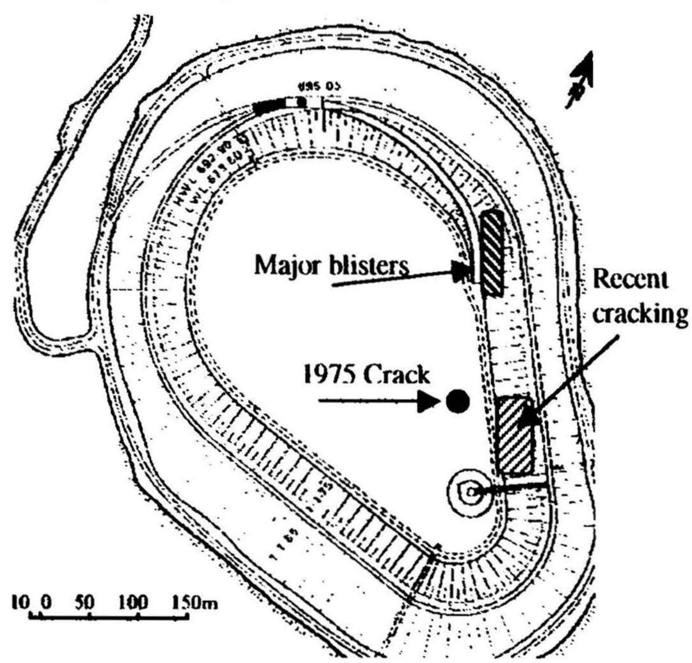


Fig. 9 Location of seepage / defects

Problems within the Dense Asphalt

Bad dispersal of asbestos fibres in the asphalt allowed small balls of matted fibre to occur. Repeated wetting followed by evaporation raised small blisters on the slope of the embankment. Hundreds of these blisters, some 20mm in diameter, were observed and treated during the first few seasons after construction. The fibre balls were gouged out and replaced by heated asphalt.

Problems between Asphalt Layers

Minute seepage of water into the junction between the binder and dense layer has resulted in hydrostatic pressure causing 100mm to 500mm blisters in the dense layer. About 10 of these blisters have occurred at the location shown on Figure 9. The source of the water which caused the pressure is not clear. These blisters never resulted in any reservoir seepage. Having attempted various repair methods, the following method has been used successfully on all the blisters.

- chisel a neat (16mm dia.) hole to the drainage layer,
- drive a hard wood plug into the dense layer leaving 40mm for an asphalt plug at the surface,
- fill with heated asphalt,
- place a flat stone on surface and cover with asphalt.

Any water present can seep to the drainage layer via the hole.

Mastic Sealing Coat

Deterioration of the mastic sealing coat on the side slopes is proportion to the amount of exposure to sunlight. The top 10m of the coat was renewed in 1993 using a sprayed bituminous emulsion in a water base.

Current Situation

Along the eastern side of the reservoir, on the upper exposed part of the lining, several cracks some 3mm wide and up to 4m long can presently be observed, see Figure 9. An opportunity for detailed inspection and perhaps correction will be afforded by an empty reservoir in July 1997.

CONCLUSIONS

1. The reservoir has performed very well in its 23 years of service to date. Movements have been negligible and within the range of surveying accuracy. This is consistent with the good standard of compaction achieved and the relatively conservative embankment fill friction angle used in design.

2. Some early seepage problems were found to be due to a defect in the lining. Since then seepage has been small.

3. Recently a slight increase in the quantity of seepage has been noted, with values up to 3 l/s. It has been shown that the rate of seepage is directly dependent on reservoir water level and appears unrelated to climatic or seasonal effects.

4. It has been shown that this increased seepage and most of the previous defects in the reservoir lining are located on the eastern and north eastern side of the reservoir. This is the location which is exposed to the evening sun during the time when the reservoir water level is low.

5. It is most likely that the increase in seepage is due to the natural aging of the bitumen in the asphaltic concrete lining layer. This process is caused by oxidation.

6. In general, however, the performance of the asphaltic concrete lining has been excellent, given that the design life of such a system would typically be 20 to 30 years. This good performance is largely due to the very tight control of construction, particularly in the hand treatment of all of the joints and also due to the diligent inspection and maintenance regime imposed by the reservoir operators.

7. Early correction of the problems of aging and a continuation of the thorough monitoring regime currently in place will be continued to ensure successful operation of the reservoir into the future.

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